Assessment of horizontal seismic coefficient for gravity quay walls by centrifuge tests

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In the simplified analysis of seismic design of gravity quay walls based on the pseudo-static approach, selection of an appropriate horizontal seismic coefficient (k_h) is important for computing the equivalent pseudo-static inertial force. However, there is no unified standard for defining k_h . There are conflicts among the existing k_h definitions regarding whether it considers (a) the effect of wall height on maximum horizontal acceleration (MHA) used in the determination of k_h or (b) the application of correction factor for k_h according to seismic performance grade. This paper evaluates the relevance of these conflicts by initially reviewing the k_h definitions in the existing codes and guidelines for port structures and then by performing a series of dynamic centrifuge tests on caisson gravity quay walls of different wall heights (5, 10 and 15 m). A review of the existing codes and guidelines has shown that the correction factor for k_h should be specified according to the seismic performance grade for the economical design of quay walls. On the other hand, based on the centrifuge tests, it was found that the MHA used in k_h calculation and the correction factor for considering seismic performance grade should reflect the effect of wall height.

KEYWORDS: centrifuge modelling; earthquakes; standards & codes of practice

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NOTATION

$a_{\max}$	reference MHA used in k_h calculation
D_a	allowable residual horizontal displacement
D_h	residual horizontal displacement
g	acceleration due to gravity
H	height of the caisson gravity quay wall model
k_h	horizontal seismic coefficient
r	correction factor on MHA in k_h calculation
T_m	mean period
T_p	predominant period

INTRODUCTION

The conventional procedures for seismic design and seismic stability evaluation of quay walls rely on a simplified method of analysis based on the pseudo-static method. In this method, Coulomb's earth pressure theory is adopted to estimate the active thrust behind the wall. The effects of earthquake on the calculation of active thrust are considered through the Mononobe–Okabe (M–O) (Okabe, 1924; Mononobe & Matsuo, 1929) and Seed–Whitman (Seed & Whitman, 1970) methods.

In the M–O method, k_h , expressed in terms of acceleration due to gravity, is used to convert the real dynamic behaviour to an equivalent pseudo-static inertial force for analysis and design. Therefore, the selection of an appropriate k_h is critical for representing the real dynamic behaviour. However, there is no unified standard for defining k_h . The port structure designers in Korea had difficulty in choosing a suitable k_h definition, as there are conflicts in defining k_h between the seismic design code (MOF, 2014) and the

seismic performance evaluation code (MLTM, 2012) of port structures. The disagreement between definitions can be related to the reference point of MHA used in the determination of k_h according to the wall height, and whether it considers the application of correction factor for k_h . Similar conflicts in k_h definitions also exist in other codes and guidelines (Werner, 1998; MOT, 1999; CEN, 2004). Thus, the definitions of k_h need to be experimentally evaluated for resolving these conflicts, so that a standard definition is possible for future design guidelines using the simplified analysis method (Bolton & Steedman, 1985; Veletsos & Younan, 1997; Iai & Sugano, 2000; Dakoulas & Gazetas, 2008; OCDI, 2009; Bellezza *et al.*, 2011; MOF, 2014; Jo *et al.*, 2017).

In this study, the key considerations to be included in k_h definition were deduced through a review of the existing codes and guidelines for port structures. Then, three dynamic centrifuge tests were performed with different wall heights (5, 10 and 15 m at prototype scale) to verify the influence of these considerations on k_h calculation.

REVIEW OF K_H DEFINITIONS

To identify the major considerations for defining k_h , the existing codes and guidelines for port structures were reviewed. The following two major concerns for defining k_h were then deduced.

- Effect of wall height on reference MHA used in k_h calculation.
- Application of correction factor on MHA used in k_h calculation according to the seismic performance grade classified by the level of acceptable damage (i.e. allowable residual horizontal displacement at the top of the wall, D_a).

An overview of the k_h definitions with respect to the aforementioned considerations is summarised in Table 1.

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Table 1. Overview of representative seismic coefficient (k_h) definitions

Codes/guidelines	Equations	Correction factor (r)		Reference MHA ($a_{\max}$)	
				H^* over 10 m	H under 10 m
CEN (2004)	$k_h = \frac{1}{r} \frac{a_{\max}}{g}$	2	Free gravity walls that can accept a displacement of up to $D_a = 300 \cdot a_{\max}$ (mm)	Average MHA	Base MHA
		1.5	Free gravity walls that can accept a displacement of up to $D_a = 200 \cdot a_{\max}$ (mm)		
		1	Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles		
MOF (2014)	$k_h = \frac{1}{r} \frac{a_{\max}}{g}$	2	$D_a = 300$ (mm)	Surface MHA	Base MHA
		1	$D_a = 0$ (mm)		
MOT (1999) and Werner (1998)	$k_h = \frac{1}{r} \left(\frac{a_{\max}}{g} \right)^{1/r}$	1	$a_{\max} < 0.2$ Special class†	Surface MHA	
		3			
		$k_h = r \frac{a_{\max}}{g}$	0.6		
MLTM (2012)	$k_h = \frac{a_{\max}}{g}$	N/A	N/A	Average MHA	

* H denotes wall height.

†Detailed in the technical standards for ports and harbour facilities in Japan (MOT, 1999).

Fig. 1. The locations of reference MHA used for the calculation of k_h

Furthermore, the locations of reference MHA ($a_{\max}$) used in calculating k_h based on each definition are shown in Fig. 1.

The k_h definitions in CEN (2004) and MOF (2014) reflect both the major concerns. To consider the effect of wall height on k_h , the location of $a_{\max}$ is specified based on the wall height of 10 m. Meanwhile, the guidelines from Werner (1998) and MOT (1999) adopted the k_h definition from the study by Noda *et al.* (1975). These guidelines apply a correction factor to MHA according to the seismic performance grade, but it does not consider the effect of wall height. On the other hand, MLTM (2012) defines k_h as the average value of the MHA along the wall height at the

backfill (average MHA), regardless of the seismic performance grade and wall height.

To visualise the influence of the above-mentioned considerations on k_h , the k_h values from k_h definitions having similar seismic performance grade (corresponding cases are indicated by shaded values in Table 1) are plotted with respect to surface MHA in Fig. 2. As $a_{\max}$ is defined differently from code to code, it is necessary to obtain the corresponding values of base MHA and average MHA from the given surface MHA for comparison. The base MHA was calculated by dividing the surface MHA by the average amplification ratio of 1.5, which was deduced from a centrifuge test by Lee *et al.* (2013). Figure 2 shows that k_h

Fig. 2. The k_h values from the existing codes and guidelines according to surface MHA

values from CEN (2004) and MOF (2014), which reflects the aforementioned considerations, are estimated to be lower as compared with other guidelines. The k_h values from MLTM (2012), which does not reflect the above considerations, are same with the conservative k_h value from CEN (2004) applicable to the ideal case of restrained wall ($D_a \cong 0$). Hence, gravity quay walls designed according to MLTM (2012) need to use the same conservative k_h value regardless of the seismic performance grade. This implies that the correction factor according to seismic performance grade should be applied to MHA used in k_h calculation for an economical design of gravity quay walls.

DYNAMIC CENTRIFUGE MODELLING

Three dynamic centrifuge tests were carried out on caisson gravity quay wall models for different wall heights to verify the effect of wall height on MHA used in the calculation of k_h and the correction factor values used in the k_h definitions. The experiments were conducted at the KOCED Geo-Centrifuge Testing Centre at KAIST using an earthquake simulator mounted on the centrifuge (Kim *et al.*, 2013a, 2013b). An equivalent shear beam (ESB) model box, which minimises the boundary effect on the soil, was used for dynamic centrifuge tests (Lee *et al.*, 2013). The centrifugal accelerations used were $40g$ for 5 and 10 m walls, and $60g$ for 15 m wall. All results are presented in prototype units.

Model structures

The caisson gravity quay wall models were designed with a k_h value of 0.13 based on a simplified quay wall design provided by MOF (2014). The models used in the experiments were made of aluminium alloy (T6061). Among the three seismic performance evaluation criteria (sliding, overturning and bearing capacity failure), sliding was set to be the first failure mode. A factor of safety of 1 was achieved in the pseudo-static force equilibrium equation with a k_h of 0.13 and other experimental conditions and the weight of the quay wall was determined. The required weight of walls was obtained by adjusting the thickness of aluminium considering the scaling law. The specifications of model structures are summarised in Table 2.

Model ground

The rubble mound and backfill were constructed by two types of dry silica sand with different mean diameters. The physical properties of the sand are presented in Table 3.

Table 2. Specifications of caisson gravity quay wall models (all values are presented in prototype scale)

	Model A	Model B	Model C
Height: m	5	10	15
Breadth: m	2.85	5.7	8.55
Length*: m	19.48	19.48	29.22
Weight: t	106	424	1436

*All the walls spanned the length of the container (i.e. 487 mm at model scale, which is 2 mm shorter than the length of the container to eliminate friction).

Table 3. Physical properties of silica sand

Properties	Silica sand (rubble mound)	Silica sand (backfill)
Unified soil classification	SP	SP
Median diameter (D_{50}): mm	0.80	0.22
Curvature coefficient (C_c)	0.98	1.11
Uniformity coefficient (C_u)	1.54	1.96
Specific gravity (G_s)	2.65	2.65
Plasticity index	NP	NP
Maximum unit weight: kN/m^3	14.19	16.45
Minimum unit weight: kN/m^3	12.87	12.44

The rubble mound was densified to a relative density of 86% by compaction for preventing overturning and bearing capacity failure. The peak friction angle of 43° was obtained from the triaxial test, and the inter-friction angle between the rubble mound and the wall was about two-third of the peak friction angle in a direct shear test (Jo *et al.*, 2014). The soil in the backfill was prepared using pluviation at a relative density of 80%.

Instrumentation

MHA values at each depth in the backfill during earthquake excitations were obtained by accelerometers buried in the soil as shown in Fig. 3. The residual horizontal displacements of the wall (D_h) were measured by potentiometers. The occurrence of overturning of the wall was evaluated by measuring D_h at the top and the bottom of the wall. To measure the subsidence of backfill and to check the bearing capacity failure of rubble mound based on the settlement of the wall, two laser sensors were used. Acceleration and D_h were positive for the active directions, and the settlement was positive for the downward direction from the surface.

Fig. 3. Schematic illustration of test instrumentation: (a) 5 m wall height, (b) 10 and 15 m wall height (all dimensions are presented in model scale)

Earthquake input motions

The Ofunato earthquake was used as an input earthquake in which the predominant period (T_p) and the mean period (T_m) (Rathje *et al.*, 1998) are 0.248 and 0.377 s, respectively. This is a common feature of earthquakes occurring in shallow bedrock conditions in Korea, which has a short-period-dominated spectral region (Manandhar *et al.*, 2017). The input earthquakes were applied incrementally in the range of 0.01–0.43g at the base of the ESB container, which represented the bedrock motion.

TEST RESULTS

MHA profile in backfill at the moment of sliding failure

To check the overturning and bearing capacity failure and to determine the initial point of sliding failure, D_h from the top and bottom of the wall and the settlements of wall and backfill were measured simultaneously and are plotted with respect to the MHA at bedrock, as shown in Fig. 4. Fig. 4 follows the information provided in Fig. 3. According to the figure, the differences between the D_h values measured at the top and bottom of the wall are small, and the settlement of the wall was minimal for increasing bedrock MHA. This shows that the overturning and bearing capacity failures did not occur and there was only a horizontal movement of the wall.

The initial point for sliding failure was defined based on the assumption that the backfill is in a state of plastic equilibrium to produce minimum active pressure. This assumption is used conventionally in the stability evaluation of sliding failure according to the M–O method. Considering this assumption, the point at which D_h starts to increase after the backfill attains the state of minimum active pressure based on Terzaghi's experiments (Terzaghi, 1941) is defined as the initial point of sliding failure. The corresponding D_h values are 5, 10 and 15 mm for 5, 10 and 15 m wall height, respectively, based on the D_h/H value of 0.001 applicable for dense sand. Also, when the backfill attains the state of minimum active pressure, the triangular soil wedge formed behind the wall fails (Das, 2015). The wedge failure can be implied from the relative settlement of

the wall and backfill, and this can alternatively be chosen as the initial point of sliding failure. The two identified initial points of sliding failure occurred at the same bedrock MHA for each case of wall height, as indicated by the arrows in Fig. 4.

To assess the two major concerns in k_h definitions for different a_{max} and correction factor, the MHA values of the backfill along the wall height at the moment of sliding failure were measured by accelerometers embedded in the soil. The MHA profiles for the three cases of wall heights at the moment of sliding failure are shown in Fig. 5.

Evaluation of two major considerations in defining k_h

The measured a_{max} values were substituted in k_h definitions having a seismic performance grade similar to the model structure (these cases are indicated by shaded values in Table 1). The obtained k_h were then compared for different codes/guidelines according to the wall height as summarised in Table 4. The percentage difference between the design k_h for the model structure (0.13) and the measured k_h was also determined.

From Table 4, it is evident that in the case of 5 and 10 m walls, the design k_h (0.13) value is similar to that obtained using half of the base MHA as suggested by CEN (2004) and MOF (2014). On the other hand, the definitions that did not consider the wall height in k_h calculation showed a big difference with the design k_h . In the case of a 15 m wall, the design k_h value is comparable with the k_h obtained from average MHA and half of the surface MHA as proposed by CEN (2004) and MOF (2014), respectively. However, for all the codes, the measured k_h values for a wall of height 15 m showed less difference as compared with the case of a wall of height equal to or less than 10 m. Additionally, the correction factor of 2.0 used in CEN (2004) and MOF (2014), which consider wall height, provides a better estimate of design k_h than those used in MOT (1999) and MLTM (2012), which does not consider the wall height.

The simplified analysis, as adopted in the existing codes and guidelines, do not consider the effect of frequency content of earthquake, damping, soil non-linearity and wall

Fig. 4. Residual horizontal displacements of wall, and settlements of wall and backfill according to bedrock MHA: (a) 5 m wall height, (b) 10 m wall height, (c) 15 m wall height

Fig. 5. MHA profile in backfill at the moment of sliding failure: (a) 5 m wall height, (b) 10 m wall height, (c) 15 m wall height

Table 4. Comparison of the design k_h value with measured k_h values from centrifuge tests

Codes/guidelines	Wall height: m	k_h definition	Design k_h	Measured k_h	Difference: %
CEN (2004)	5	$k_h = \frac{1}{2} \frac{(\text{basePGA})}{g}$	0.130	0.114	-12
	10	$k_h = \frac{1}{2} \frac{(\text{averagePGA})}{g}$		0.133	2
	15			0.112	-14
MOF (2014)	5	$k_h = \frac{1}{2} \frac{(\text{basePGA})}{g}$	0.130	0.114	-12
	10	$k_h = \frac{1}{2} \frac{(\text{surfacePGA})}{g}$		0.133	2
	15			0.144	11
MOT (1999) and Werner (1998)	5	$k_h = 0.6 \frac{(\text{surfacePGA})}{g}$	0.130	0.305	135
	10			0.401	208
	15			0.173	33
MLTM (2012)	5	$k_h = \frac{(\text{averagePGA})}{g}$	0.130	0.385	196
	10			0.378	191
	15			0.223	72

translation. The limitations of the pseudo-static method based on M–O were assessed by comparing the differences between pseudo-static and true dynamic behaviours in centrifuge tests by many researchers (Nakamura, 2006; Al Atik & Sitar, 2010; Brandenberg *et al.*, 2015; Hushmand *et al.*, 2016; Wagner & Sitar, 2016; Jo *et al.*, 2014, 2017). In this regard, the definition of k_h could be further improved in future studies. In addition, the influence of excess pore water pressure developed in the backfill during earthquake should be evaluated for port structures (Zeng & Steedman, 1993; Iai *et al.*, 1998; Kim *et al.*, 2005; Lee, 2005).

CONCLUSIONS

In this study, the existing codes and guidelines for port structures were reviewed and dynamic centrifuge tests were performed to identify and evaluate the two major considerations in k_h definition used in the simplified analysis method of designing gravity quay walls. It was found that the correction factor according to the seismic performance grade and the effect of wall height should be considered on MHA used in k_h calculation. Further, correction factors for the MHA used in k_h definition should be appropriately derived for different wall heights. This research has provided an important direction for arriving at a unified definition of k_h to be used in seismic codes and guidelines for the simplified analysis of gravity quay walls.

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